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SUBBALLAST REQUIREMENT STUDY

DYNAMIC TRIAXIAL TESTING PROGRAM

PRESENT STATUS

Prepared for

CN RAIL RESEARCH CENTRE
Montreal, Quebec

and for

TRANSPORT CANADA
TRANSPORTATION DEVELOPMENT CENTRE

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Un sommaire en français de ce rapport est inclus avant la Table des Matières.

The contents of this report reflect the views of the contractor and not necessarily the official views or policies of the Transportation Development Centre (TDC) of Transport Canada.

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17. Abstract

In order to obtain suitable material parameters for use in a railroad track granuar depth design method, it is necessary to adopt a relevant laboratory test. In this case the dynamic triaxial test was chosen to evaluate subgrade soil properties. Composite soil samples from the alignment of Calder Yard Bypass, Edmonton, Alberta, were subjected to a series of dynamic triaxial tests. Also, standard Proctor compaction tests and static triaxial tests were performed.

Preliminary results show that threshold "stress" and "strain" levels exist. The general behaviour of the composite soil appears to follow the trends described in the British Rail granular depth design method.

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17. Résumé

La mise en évidence des propriétés physiques et mécaniques des sols support, qu'il est intéressant de connaître pour le calcul de l'épaisseur des couches granulaires devant composer l'infrastructure d'une voie ferrée, nécessite des essais en laboratoire adaptés au but que l'on s'est fixé. Dans le cas qui nous intéresse, il a été décidé de faire appel aux essais triaxiaux dynamiques, complétés d'essais Proctor et d'essais triaxiaux statiques. Les échantillons de sols ont été prélevés dans la partie rectiligne de la voie d'évitement de la gare de triage Calder à Edmonton (Alberta).

Les résultats préliminaires ont mis en évidence les niveaux critiques de contrainte et de déformation. Par ailleurs, le comportement général du sol considéré semble correspondre à celui pris en compte dans la méthode britannique de calcul de l'épaisseur des couches granulaires des infrastructures ferroviaires.

18. Mots-clés	19. Diffusion
Voie ferrée, chemin de fer, ballast, épaisseur, couche granulaire, propriétés des sols, contrainte, déformation, essai triaxial dynamique, essais de compression	Exemplaires disponibles au Centre de développement des transports

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DYNAMIC TRIAXIAL TESTING PROGRAM

EXECUTIVE SUMMARY

Written by CN RAIL RESEARCH

This report, prepared by the Geotechnical Research Centre, McGill University, under contract to CN Rail Research, documents results from laboratory soil tests conducted to obtain relevant soil parameters.

A sine pulse loading wave was chosen as being a fair representation of the subgrade stress wave resulting from moving axle loads. Preliminary test results are presented herein. Threshold stress and strain levels are identified for the composite soil sample. More testing is required in order to develop a "cumulative damage" relationship.

This report is the first in a series of reports describing work undertaken by CN Rail Research in an effort to develop granular depth design charts for railroad track.

PROGRAMME D'ESSAIS TRIAXIAUX DYNAMIQUES

SOMMAIRE ADMINISTRATIF

Rédigé par le Centre de recherche sur le rail du CN

Ce rapport, réalisé par le Centre de recherche géotechnique de l'université McGill pour le compte du CN, présente les résultats des essais en laboratoire exécutés pour dégager certaines caractéristiques intéressantes des sols considérés.

Il a été décidé de soumettre les échantillons de sols à un régime de charge cyclique sinusoidale tenu pour être une bonne représentation des contraintes alternées induites dans le sol support d'une voie ferrée par le passage d'essieux chargés. Les résultats préliminaires des essais mettent en évidence des valeurs critiques de contrainte et de déformation et la nécessité de procéder à d'autres essais pour connaître l'effet cumulatif des charges sur la tenue du sol support.

Ce rapport est le premier d'une série concernant les travaux du Centre de recherche sur le rail du CN visant à mettre au point une méthode de calcul de l'épaisseur des couches granulaires d'une voie ferrée.

DYNAMIC TRIAXIAL TESTING PROGRAM

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DYNAMIC TRIAXIAL TESTING PROGRAM
PRESENT STATUS

SCOPE OF TESTING PROGRAM

A series of soil samples from Test Pits 2A, 14, 16, 9, and 25 were to be utilized in a triaxial testing program to provide material properties which could be used in the British Rail Ballast Thickness design method. Two composite samples were to be prepared by mixing samples from Test Pits 2A, 14 and 16 (herein called Soil "A") and samples from Test Pits 9 and 25 (herein called Soil "B"). The composite samples were to be subjected to the standard soil identification tests and Standard Proctor compaction tests were to be conducted.

The initial triaxial tests were to be conducted on compacted specimens which represented relatively poor field conditions (i.e. 1) compaction water content greater than Standard Proctor optimum water content, 2) specimens to be fully soaked and saturated with back pressure, 3) undrained test conditions, 4) lateral principal stress (σ_3) limited to 2 psi which would be equivalent to about 24 inches of granular material below the base of rail. The importance of these initial constraints was to be investigated by subsequent testing as time permitted.

PREPARATION AND PROPERTIES OF COMPOSITE SAMPLES

Composite soil "A" was prepared by cutting the intact lumps of material from Test Pits 2A, 14 and 16 into small pieces and blending to form a large sample. The entire sample was passed through a No. 4 sieve at the "as received" water content. The sample was throughly re-mixed and a

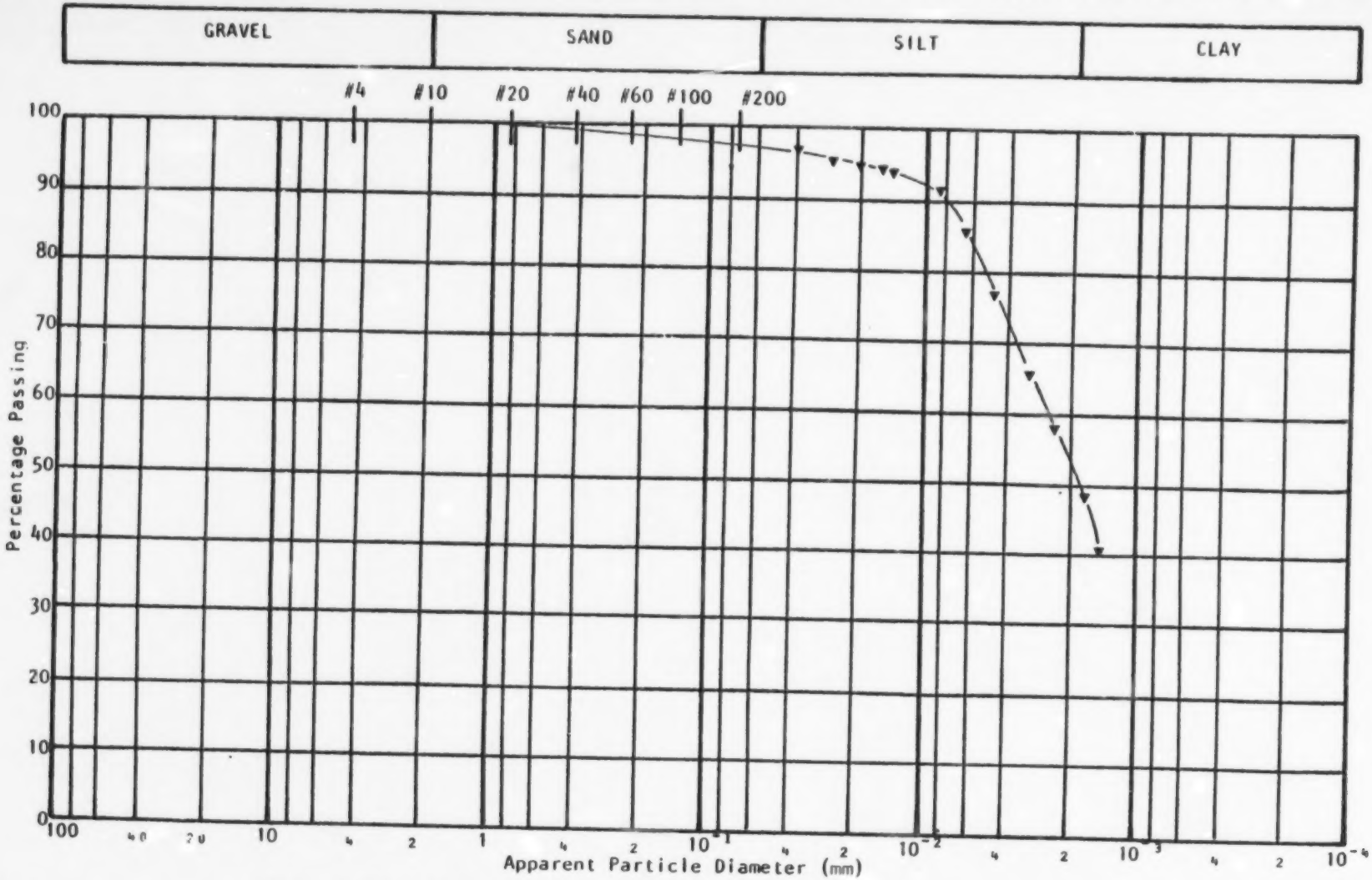
representative portion was used in the determination of the basic engineering classification tests. The resulting data are given in Figure No. 1.

Composite soil "B" was prepared from Test Pit samples 9 and 25 as described for soil "A". The engineering classification data are given in Figure No. 2. It should be noted that soil "B" contains a significantly higher proportion of clay size materials than soil "A" and that this is reflected in higher values of liquid and plastic limit.-

Mineralogical identification of the fine grained fraction has not been completed at the time of reporting.

COMPACTION TESTS ON COMPOSITE SAMPLES

Standard Proctor Compaction tests were conducted on composite soils "A" and "B". The test data are given in Figure No. 3.



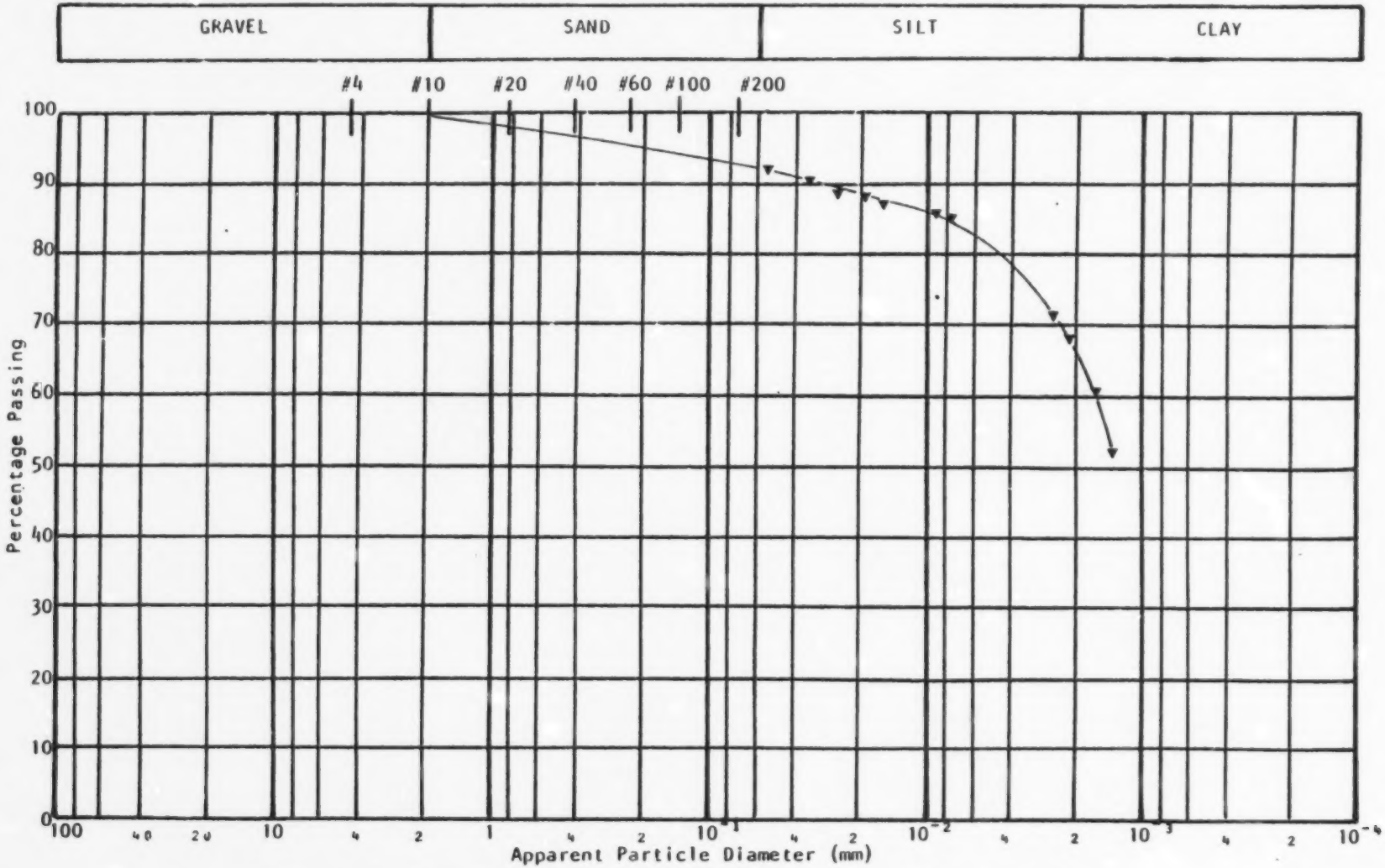
Natural Water Content = 36%

Specific Gravity = 2.77

Liquid Limit = 70%

Plastic Limit = 28.5%

FIGURE NO. 1: ENGINEERING IDENTIFICATION OF COMPOSITE SOIL "A".



Natural Water Content = 37%

Liquid Limit = 92%

Specific Gravity = 2.78

Plastic Limit = 35%

FIGURE NO. 2: ENGINEERING IDENTIFICATION OF COMPOSITE SOIL "B".

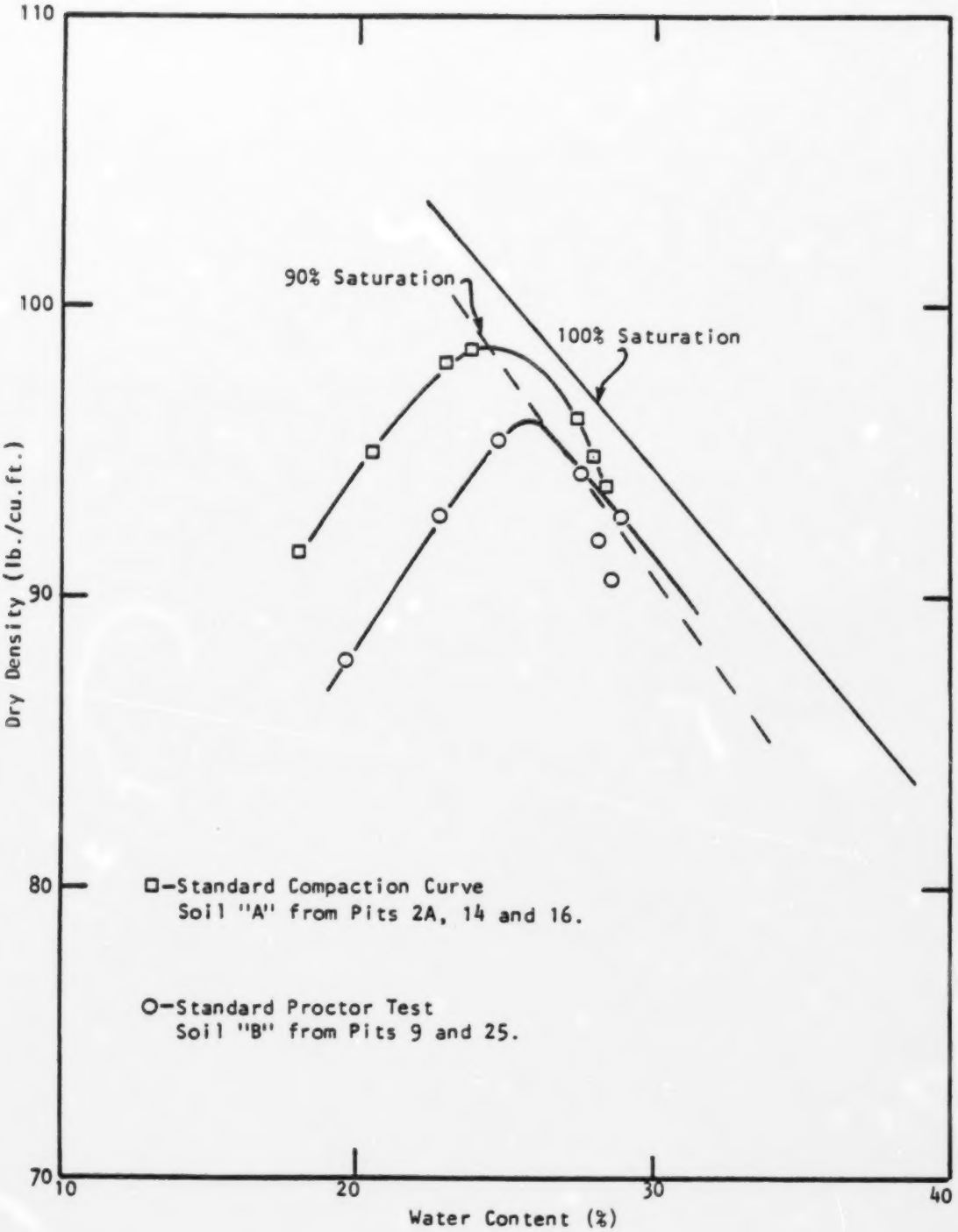


FIGURE NO. 3: STANDARD PROCTOR MOISTURE-DENSITY RELATIONSHIP FOR COMPOSITE SOILS "A" AND "B".

PREPARATION OF TRIAXIAL TEST SPECIMENS

In order to simulate a poor subgrade condition triaxial specimens were compacted a water content above the Standard Proctor optimum water content. Water contents of 28% and 28.5% were chosen for soils "A" and "B", respectively. The triaxial test specimens were compacted in a mold formed by connecting two standard compaction molds together to form a soil cylinder 4 inches in diameter by 8 inches high. The specimen was compacted according to Standard Proctor criteria. Since the specimen is made up of a total of six sub-layers, the top of each sub-layer was scarified to a depth of about 1/8" prior to compaction of the next layer to reduce the discontinuity between the layers.

The compacted specimens were surrounded with side filter drains to reduce pore pressure equlization time and installed in triaxial cells with provision for both top and bottom drainage. The cells were filled with water only up to the top of the specimen to provide a compressible air layer to allow dynamic motion of the loading piston without significantly changing the cell pressure. An external volume change buret was connected to the pore water drainage lines and the water level in the buret was maintained equal to the water level in the triaxial cell. In this way the hydrostatic pressure distribution due to the self-weight of water in the cell was balanced by the hydrostatic pore pressure distribution within the soil. This factor is important because of the very small confining pressures to be applied in these tests.

STATIC TRIAXIAL TEST RESULTS

Back pressure was applied in 5 psi increments up to 30 psi or until a "B" pore pressure parameter of about .95 was reached. Then a 2 psi isotropic

consolidation stress was applied. The specimens were then loaded in a constant deformation rate testing machine. The deformation rate was adjusted to provide a specimen strain rate of about 1.5% per hour. The consolidation data indicated that this rate was sufficiently low to allow 95% equalization of pore pressure, which is the standard testing criteria.

The test parameters for specimens prepared from composite sample "A" are given in Table 1. Figures 4 and 5 show data for confining pressures of 2 and 4 psi, respectively. It should be noted that the specimens failed plastically and well developed shear planes were observed. The pore pressures reach a peak initially and then decline which is characteristic of compacted materials tested at low confining pressures. The stress path diagrams for these tests are presented in Figure No. 6 and the computed values of the Mohr shear strength parameters are given in the Figure.

The test parameters for specimens prepared from composite sample "B" are given in Table 1. Figure Nos. 7, 8 and 9 present the relationships between axial stress, axial strain and pore pressure for confining pressures of 2 psi, 2 psi and 4 psi, respectively. As for soil "A", the pore pressures tend to decrease following failure. Figure No. 10 shows the stress path diagrams for these tests. It should be noted that the water content values for these tests are much higher than for the case of soil "A" tests which reflects the higher swelling potential of the clay fraction in the composite "B" soil.

TABLE 1: STATIC TRIAXIAL TEST PARAMETERS FOR SPECIMENS PREPARED FROM COMPOSITE SOILS "A" AND "B".

TEST NUMBER	SOIL TYPE	TEST CONDITION	CONSOLIDATION PRESSURE (psi)	BACK PRESSURE (psi)	PEAK CYCLIC DEVIATOR STRESS (psi)	NO. OF CYCLES	CUMULATIVE STRAIN (%)	RANGE OF ELASTIC STRAIN (%)	RANGE OF DYNAMIC MODULUS (psi)	RANGE OF PORE PRESSURE (psi)	INITIAL WATER CONTENT (%)	FINAL WATER CONTENT (%)	DRY DENSITY (lbs/cu.ft.)	REMARKS
1	A	Undrained	2.0	30	7.8	-	25.0	-	-	+ 1.5	28.0	32.3	93.7	Sample Failed
9	A	Undrained	4.0	35	10.4	-	24.0	-	-	+2.25 → -0.6	28.5	33.2	94.44	Sample Failed
11	B	Undrained	2.0	30	3.05	-	4.0	-	-	+1.14 → +.48	28.5	42.97	90.36	Sample Failed
13	B	Undrained	4.0	30	5.2	-	2.0	-	-	+1.9 → +0.7	28.5	38.07	90.12	Sample Failed
15	B	Undrained	2.0	30	3.6	-	2.4	-	-	+1.24 → -0.2	28.5	40.5	90.77	Sample Failed

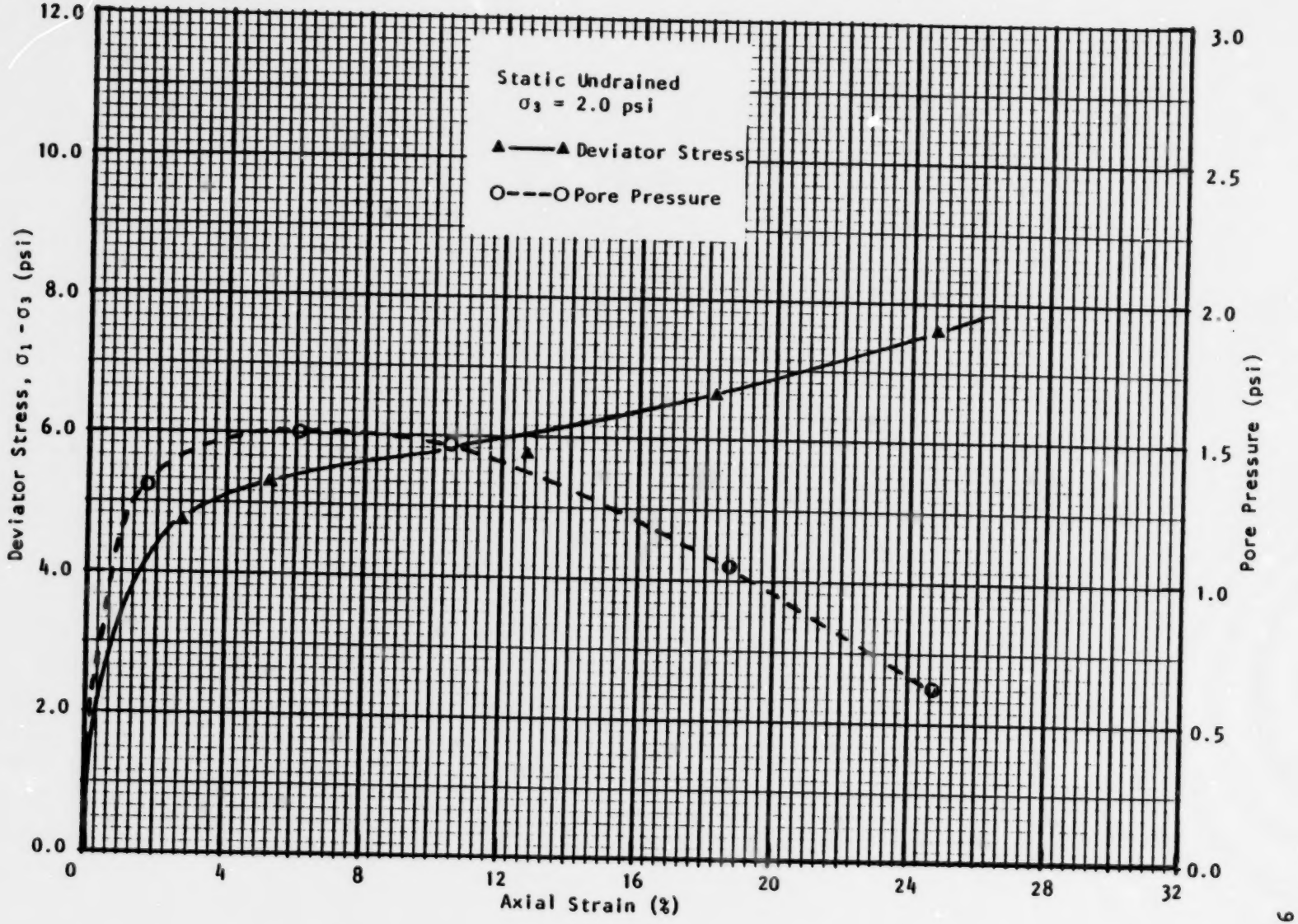


FIGURE NO. 4: DEVELOPMENT OF STRAIN AND PORE PRESSURE WITH INCREASING DEVIATOR STRESS UNDER CONSTANT DEFORMATION RATE FOR TEST NO. 1 (COMPOSITE SAMPLE "A").

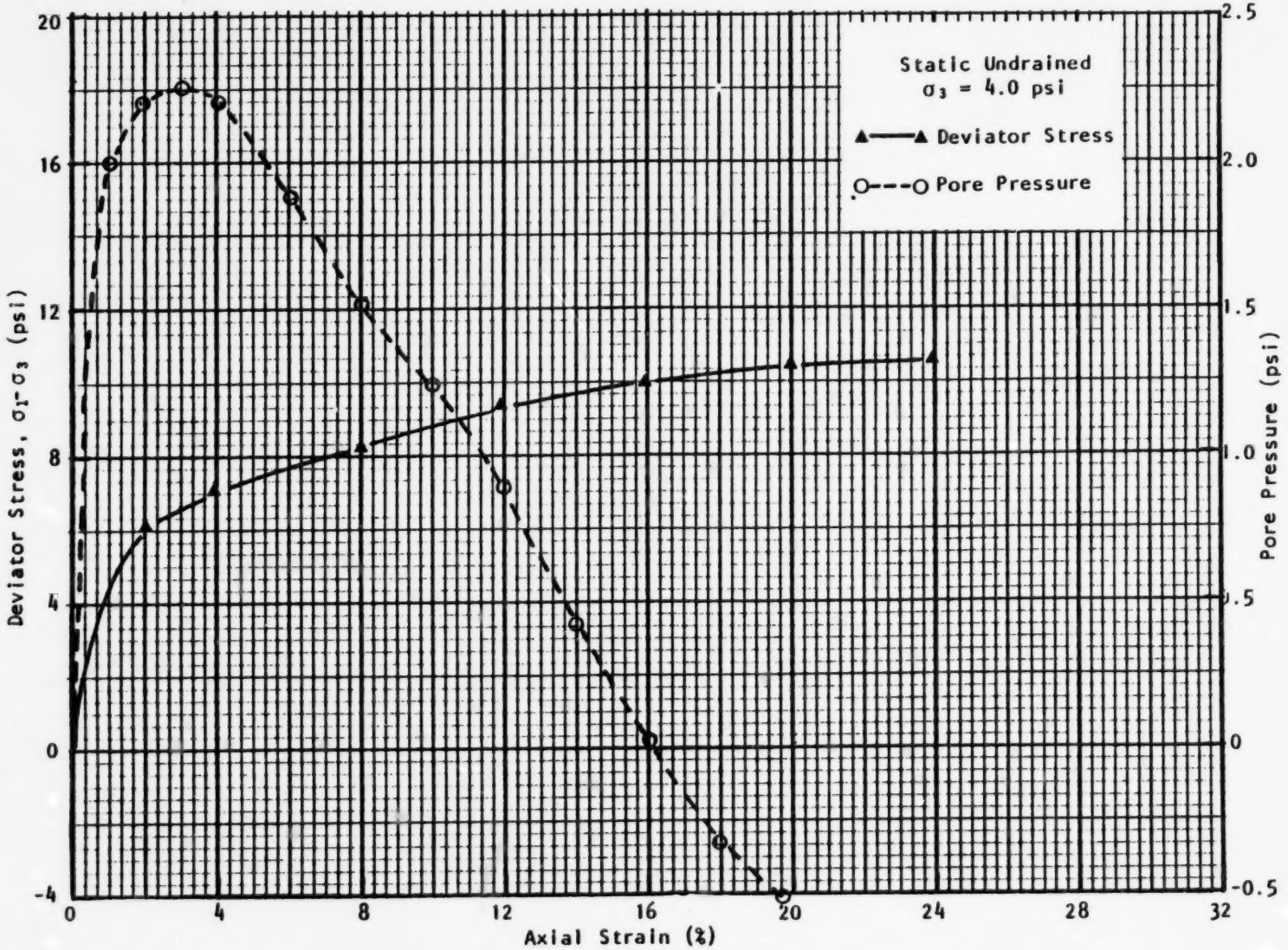


FIGURE NO. 5: DEVELOPMENT OF STRAIN AND PORE PRESSURE WITH INCREASING DEVIATOR STRESS UNDER CONSTANT DEFORMATION RATE FOR TEST NO. 9 (COMPOSITE SAMPLE "A").

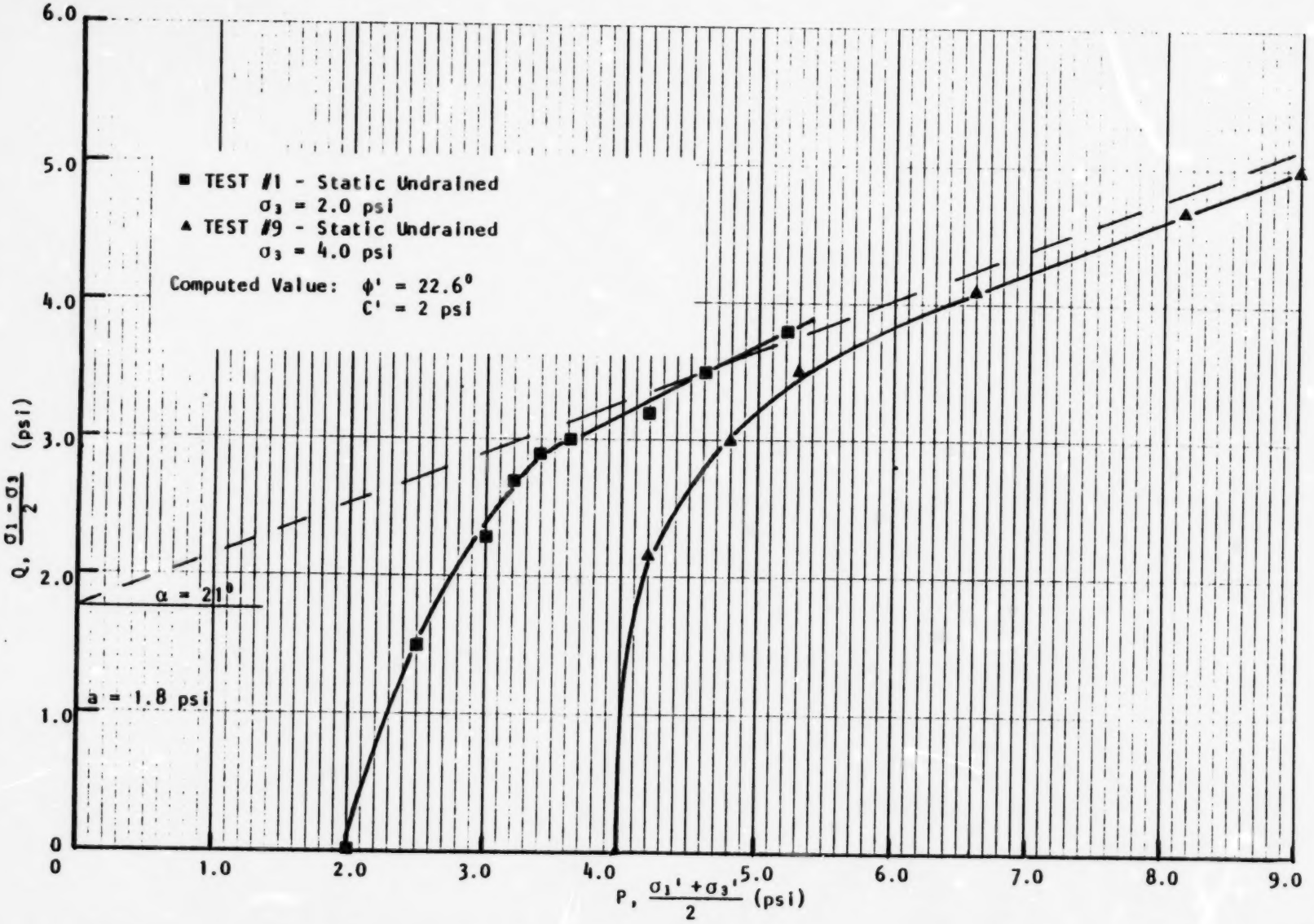


FIGURE NO. 6: STRESS PATH DIAGRAM FOR STATIC TESTS PERFORMED ON COMPOSITE SOIL "A".

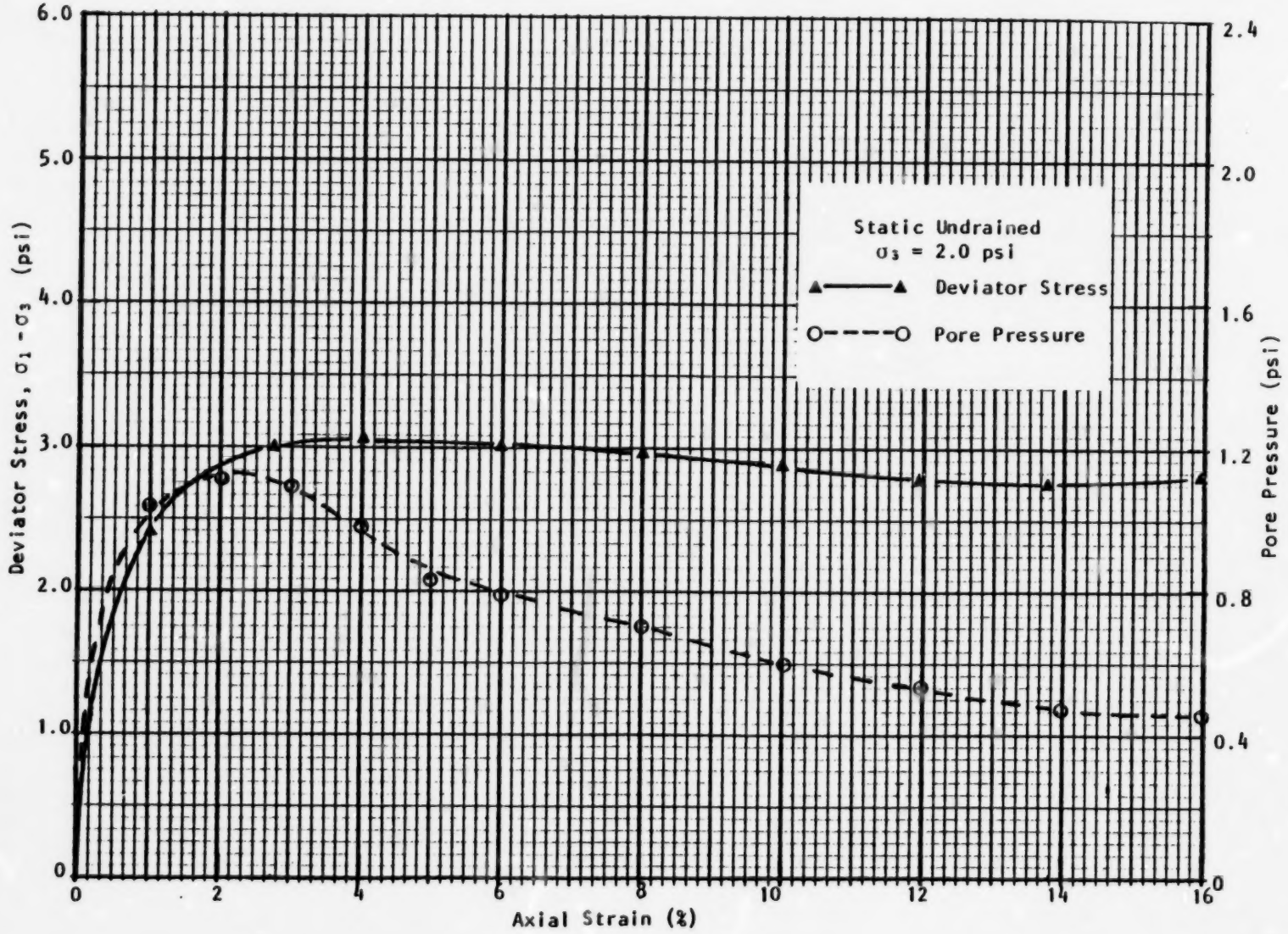


FIGURE NO. 7: DEVELOPMENT OF STRAIN AND PORE PRESSURE WITH INCREASING AXIAL STRESS UNDER CONSTANT DEFORMATION RATE FOR TEST NO. 11 (COMPOSITE SAMPLE "B").

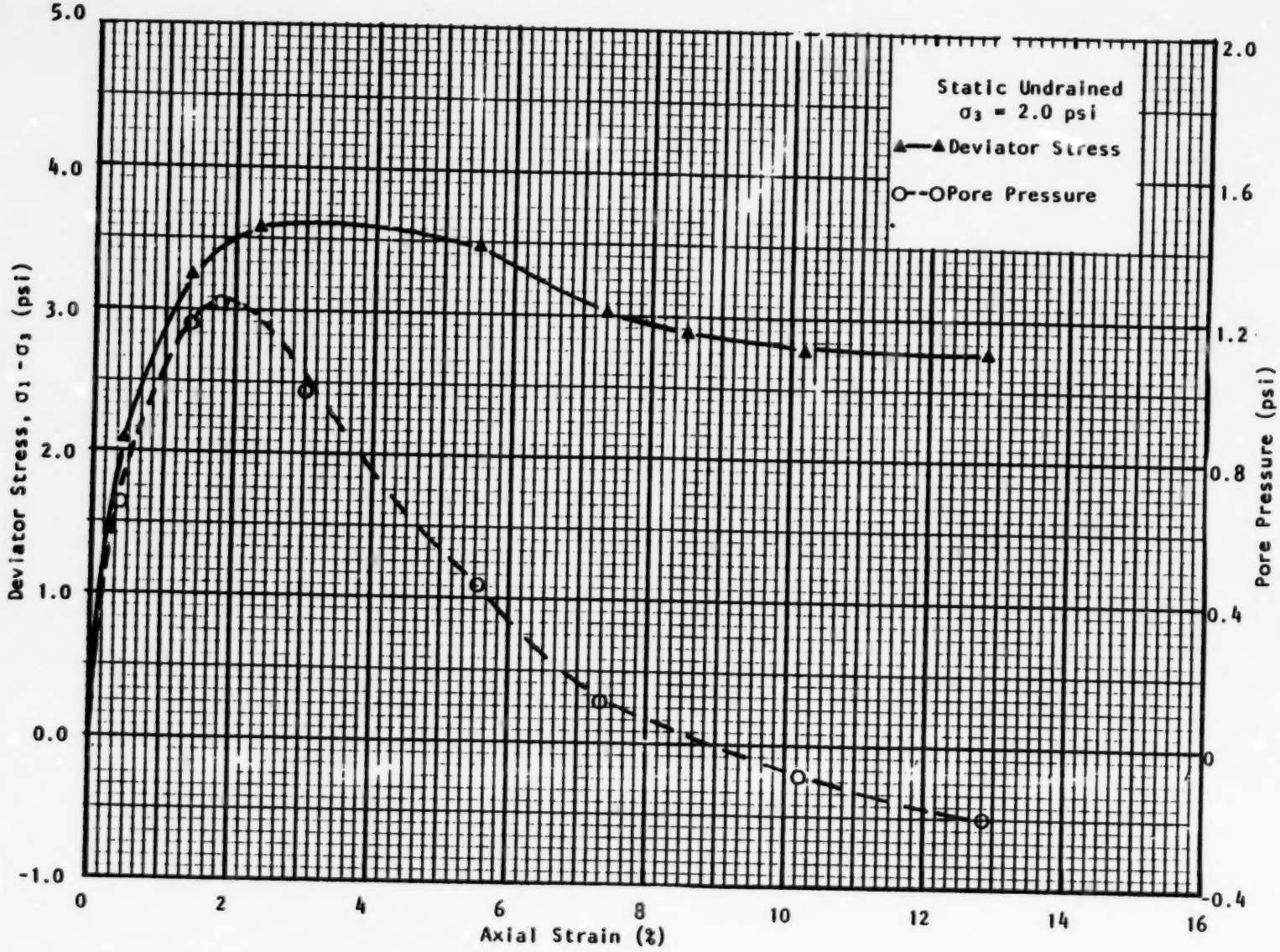


FIGURE NO. 8: DEVELOPMENT OF STRAIN AND PORE PRESSURE WITH INCREASING AXIAL STRESS UNDER CONSTANT DEFORMATION RATE FOR TEST NO. 15 (COMPOSITE SAMPLE "B").

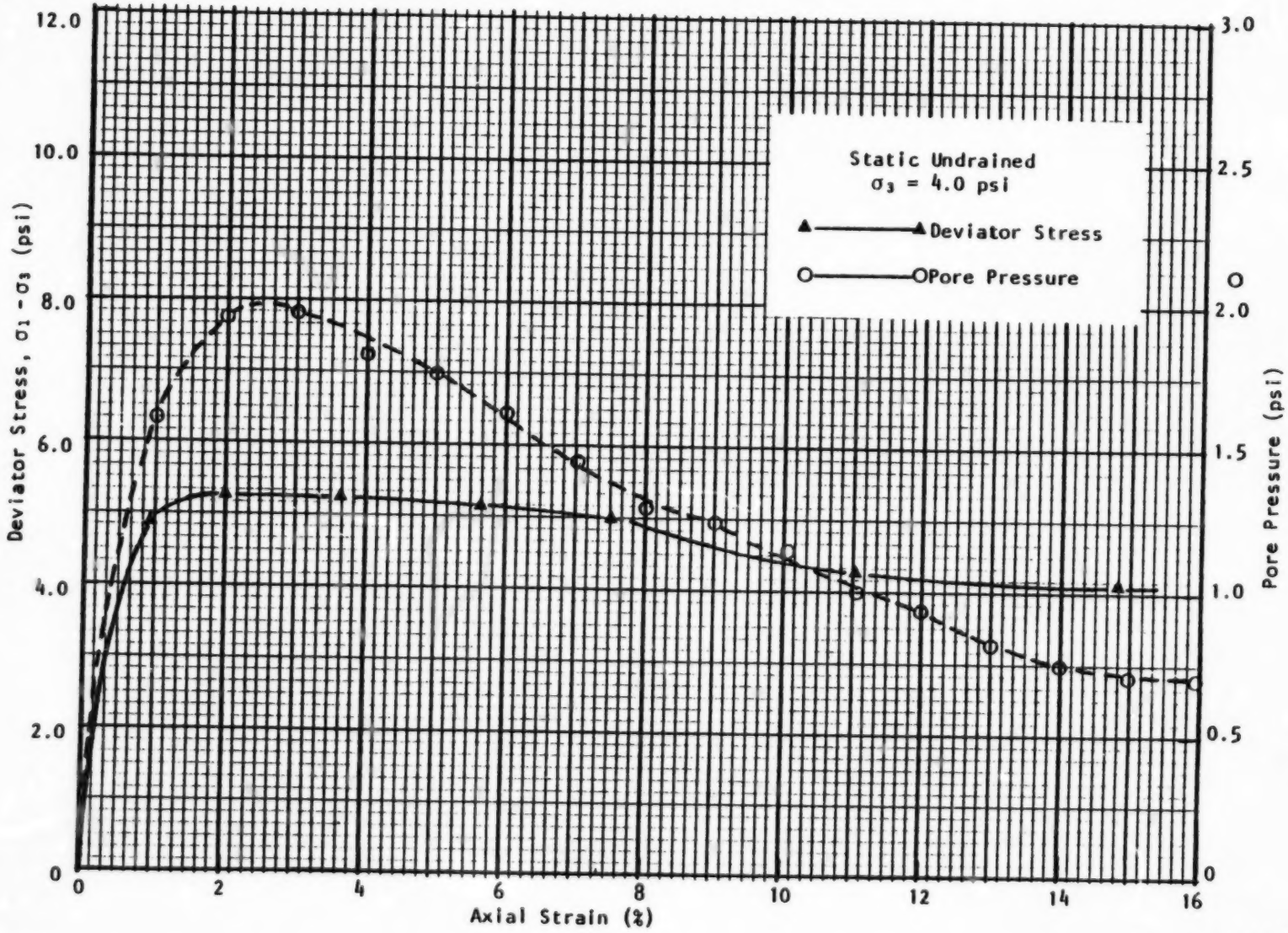


FIGURE NO. 9: DEVELOPMENT OF STRAIN AND PORE PRESSURE WITH INCREASING AXIAL STRESS UNDER CONSTANT DEFORMATION RATE FOR TEST NO. 13 (COMPOSITE SAMPLE "B").

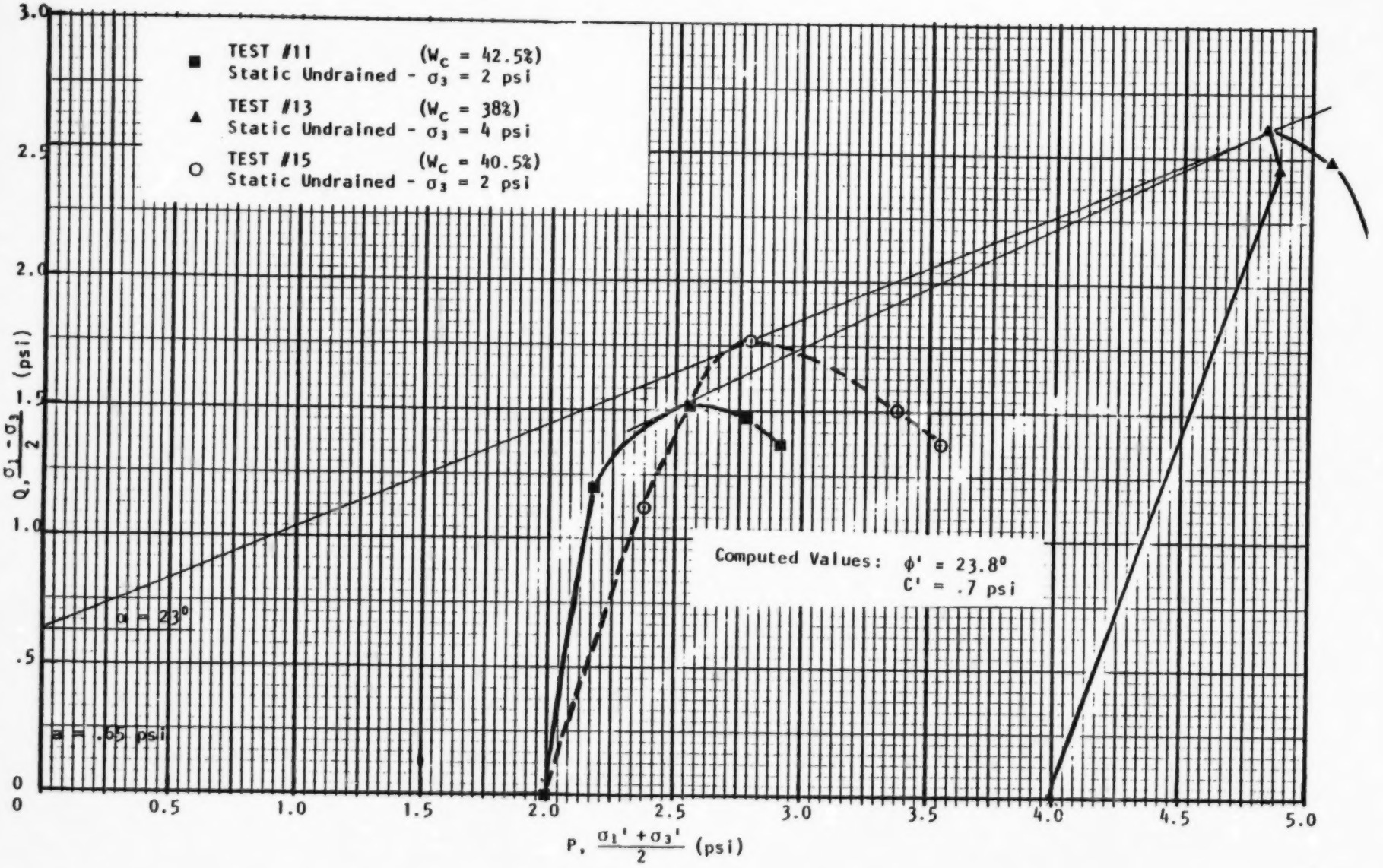


FIGURE NO. 10: STRESS PATH DIAGRAM FOR STATIC TESTS PERFORMED ON COMPOSITE SOIL "B".

DYNAMIC TRIAXIAL TEST RESULTS FOR COMPOSITE SOIL "A"

The dynamic triaxial tests were performed using a closed-loop electrohydraulic servo system. A function generator supplies a reference voltage to a servo valve controller. The servo valve admits or releases oil from a hydraulic loading ram which is connected through a load cell to the triaxial cell piston. The controller compares the reference voltage from the function generator with the load cell output and adjusts the position of the servo valve such that the load cell output follows the form of the reference voltage. A sine function was used in all tests to date. The stress applied to the sample ($\sigma_1 - \sigma_3$) varied between zero and the preset peak deviator stress.

Table 2 shows the parameters for dynamic tests conducted to date on specimens prepared from composite sample "A". Tests No. 3 and No. 4 were essentially calibration tests. These tests are not discussed in detail at this time because it can be shown that a cyclic stress history at a given amplitude affects the strain rate at a higher stress amplitude. Proper interpretation of these tests must await further testing to develop a "cumulative damage" relationship.

Figure No. 11 shows the relationships between cumulative axial strain and number of load cycles for specimens prepared from composite sample "A". There are several features to note here:

1. With the exception of Test No. 7 and No. 16, the slope of the cumulative strain versus log number of cycles is relatively constant. This is not anticipated from the British Rail Data.

2. Test No. 16, which was repeatedly subjected to a set of 500 cycles and then drained for 30 minutes, appears to follow a somewhat dif-

TABLE 2: DYNAMIC TRIAXIAL TEST PARAMETER FOR SPECIMENS PREPARED FROM COMPOSITE SAMPLE "A".

TEST NUMBER	SOIL TYPE	TEST CONDITION	CONSOLIDATION PRESSURE (psi)	BACK PRESSURE (psi)	PEAK CYCLIC DEVIATOR STRESS (psi)	No. of CYCLES	CUMULATIVE STRAIN (%)	RANGE OF ELASTIC STRAIN (%)	RANGE OF DYNAMIC MODULUS (%)	RANGE OF PORE PRESSURE (psi)	INITIAL WATER CONTENT (%)	FINAL WATER CONTENT (%)	DRY DENSITY (lbs./cu.ft.)	REMARKS
2	A	Undrained	2.0	30	4.0	6400	5.65	0.49 → 0.50	800 → 820	+.04 → -0.21	27.7	35.1	92.65	Sample Failed
3	A	Drained	0.0	30	1.6	30000	0.04	.065 → .068	2420 → 2450	Drained Conditions, P.P. Measurements Non-applicable				
					3.2	20000	0.07	0.14 → 0.12	2250 → 2650					
					4.4	22000	0.29	0.2 → 0.21	2250 → 2050		28.3	34.0	94.6	
					5.6	13000	3.25	0.36 → 0.41	1400 → 1500					
					6.4	2400	6.8	0.41 → 0.52	1200 → 1550					Sample Failed
4	A	Undrained	2.0	30	3.2	13000	.078	0.09 → 0.05	3500 → 6300	+.4 → +.33				
					5.6	27000	.078	0.26 → 0.3	2150 → 1850	+.58 → +.3	27.5	32.0	94.7	
					7.2	18000	2.65	0.54 → 0.58	1250 → 1500	+0.6 → +.7				
					8.0	1080	6.3	0.86 → 1.2	800 → 900	0.0 → -1.3				Failed
5	A	Undrained	2.0	30	6.4	130000	5.7	0.66 → 0.50	970 → 1280	+.94 → -0.58	27.4	32.6	94.9	Failed
6	A	Undrained	2.0	40	5.6	430000	4.5	.3 → .345 → .275	1600 → 2200	+1.12 → 0.0	28.0	33.3	93.77	Not Failed
7	A	Undrained	2.0	40	7.2	7800	11.25	.4 → .75 → .69	960 → 1040	+.73 → -0.28	28.5	32.2	93.6	Failed
8	A	Undrained	2.0	35	6.0	780000	1.8	0.36 → 0.17	1650 → 3500	+1.04 → -1.12	27.52	30.77	94.9	Not Failed
10	A	Undrained	2.0	35	6.0	165000	3.3	0.48 → 0.27	1350 → 2200	+1.08 → +0.24	27.8	32.48	94.43	Not Failed
16	A	Undrained during cycling -Drainage in between cycling for 30 min	2.0	40	6.6	21000	14.5	.36 → .62 → .48	1100 → 1700	+1.2 → -.34	26.5	32.2	96.0	Failed

ferent strain history, but failure occurs at a number of cycles which is constant with the peak deviator stress level.

3. A reduction of peak deviator stress by 50% (Test No. 4 versus test No. 7) produces very small cumulative strain values.

4. British Rail procedures would suggest that a stress level of about 6 psi would lead to an equilibrium situation at about 1,000,000 cycles and further cumulative strain would not be anticipated.

Figure No. 12 shows that the threshold elastic strain is about 0.5% and that failure is not anticipated if strain levels are maintained below this limit. It should be noted that elastic strain values decrease with number of load cycles. This behavior may be related to gradual reduction in pore pressure following initial increase shown in Figure No. 13. The reduction in pore pressure is consistant with the dilatant nature of the material as shown in the static tests. Values of dynamic modulii are given in Figure No. 14. These values are the simple ratio of dynamic stress (psi) to dynamic strain (inches/inch).

Figure No. 13 shows that the periodically drained test (No. 16) exhibits rather unusual pore pressure behavior. The pore pressure values are the maximum pore pressure developed during the undrained load cycling phase. During the 30 minute drained phase, in which there was no axial load applied to the sample ($\sigma_1 - \sigma_3 = 0$) the volume change buret indicated that the specimen was continuously increasing in volume. The expression of this swelling potential seems to be enhanced by the cyclic load application. However, after about 10,000 load applications when cumulative strain has become important the expected drop in pore pressure associated with onset of shear is observed.

Figure No. 15 shows the relationship between strain rate and number

of load cycles. Whereas it is quite clear that Test No. 7 (7.2 psi) shows the highest cumulative strain rate at a given number of load cycles, it is apparent that very significant cumulative strain occur in the first load cycles. Further analysis of test data in this manner may lead to the formulation of a "cumulative damage" low which relates strain rate to the number of load cycles at particular stress level and includes the influence of previous cycling history.

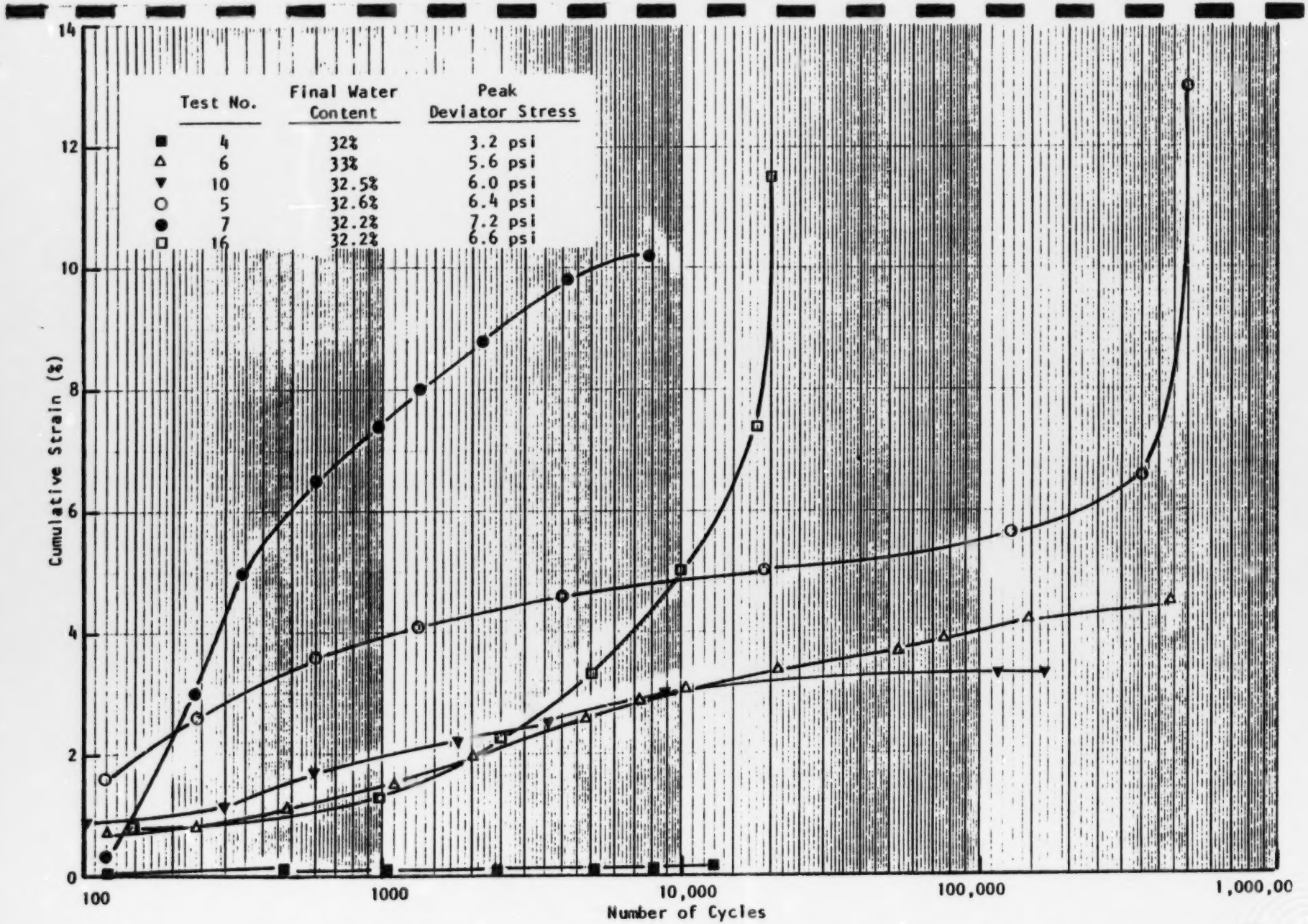


FIGURE NO. 11: RELATIONSHIPS BETWEEN CUMULATIVE AXIAL STRAIN AND NUMBER OF LOAD CYCLES FOR SPECIMENS FROM COMPOSITE SAMPLE "A".

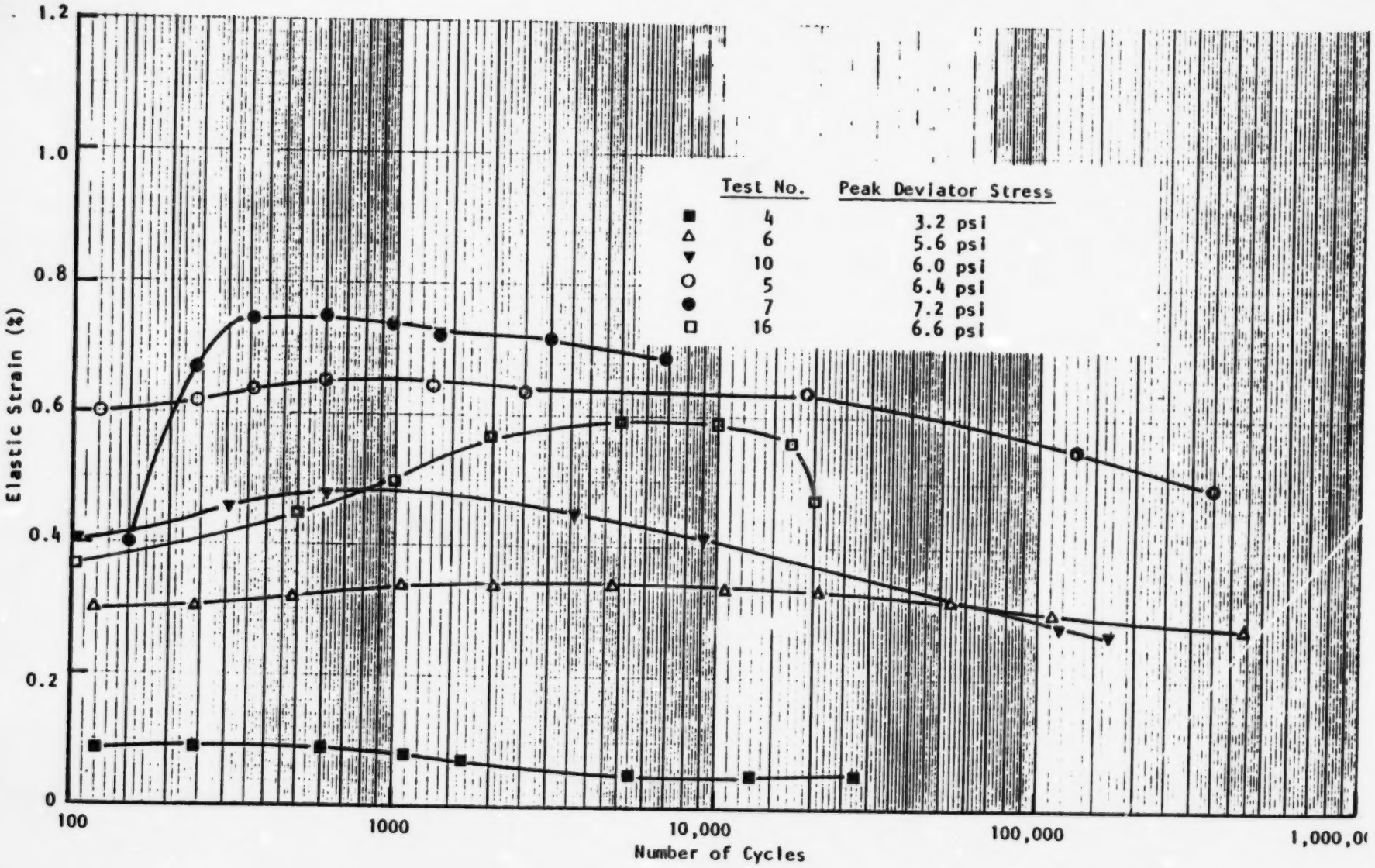


FIGURE NO. 12: RELATIONSHIPS BETWEEN AXIAL ELASTIC STRAIN AND NUMBER OF LOAD CYCLES FOR SPECIMENS FROM COMPOSITE SAMPLE "A".

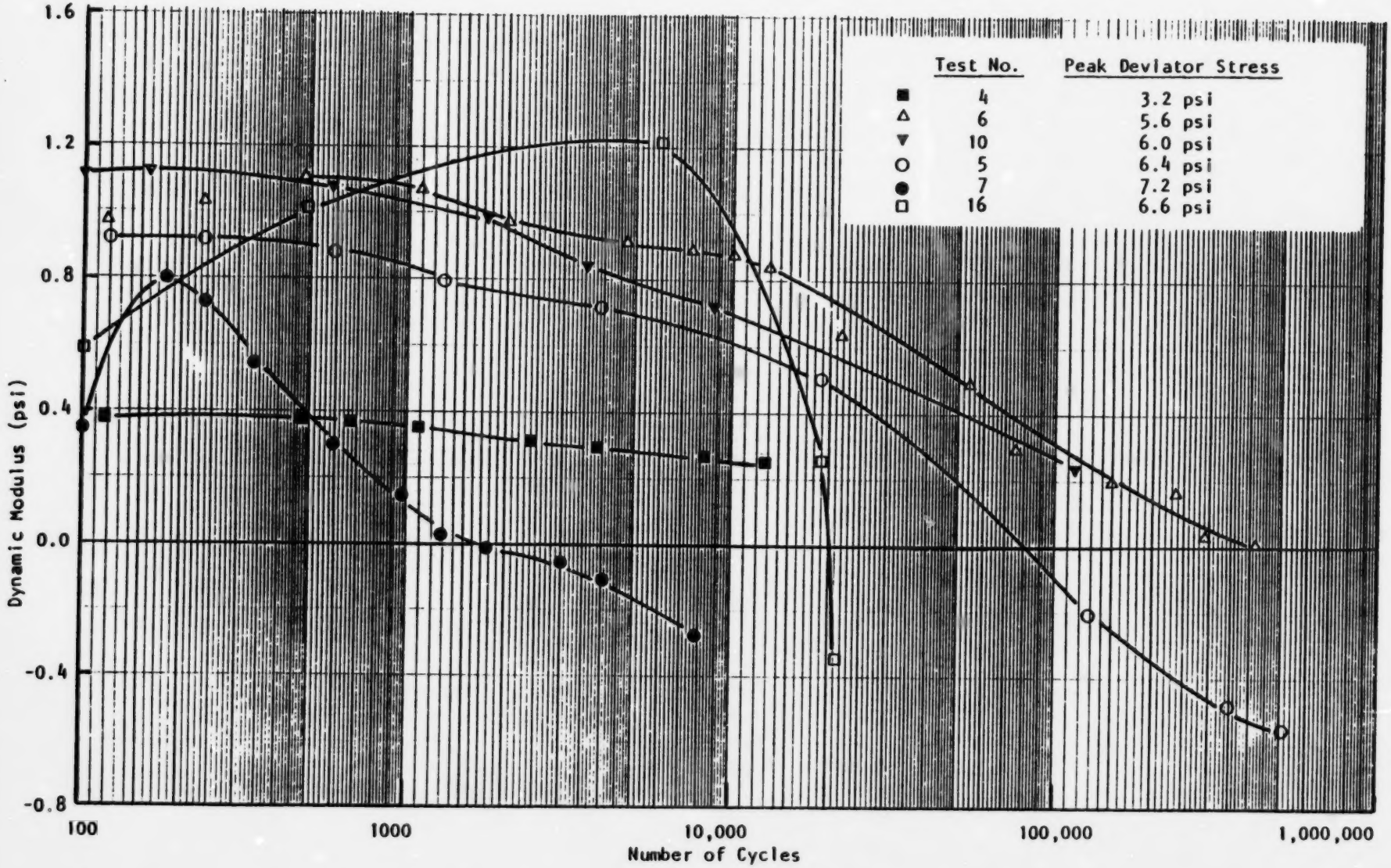


FIGURE NO.13: VARIATION IN PORE PRESSURE WITH NUMBER OF LOAD CYCLES FOR SPECIMEN FROM COMPOSITE SAMPLE "A".

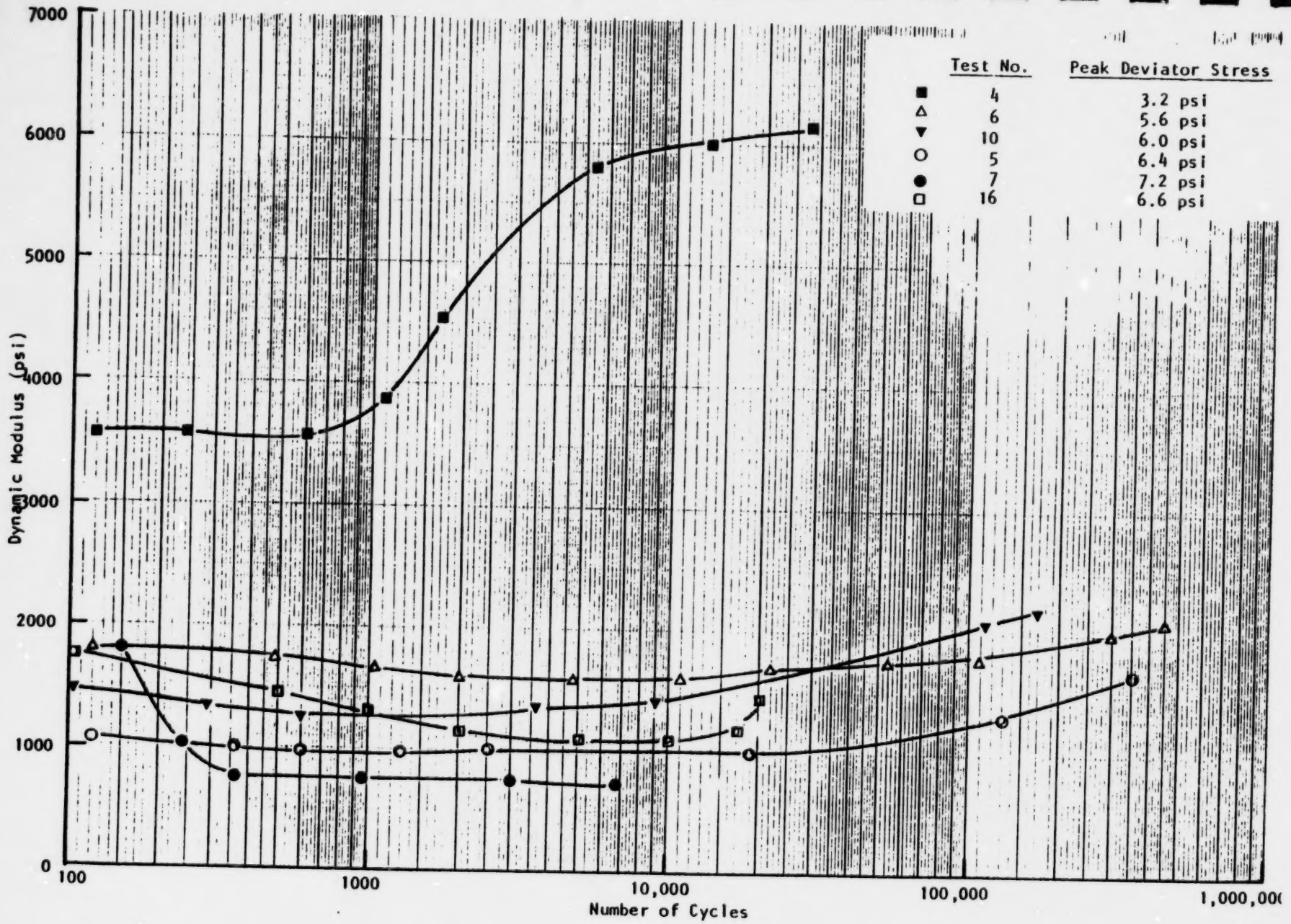


FIGURE NO. 14: VARIATION IN DYNAMIC MODULUS WITH NUMBER OF LOAD CYCLES FOR SPECIMENS FROM COMPOSITE SAMPLE "A".

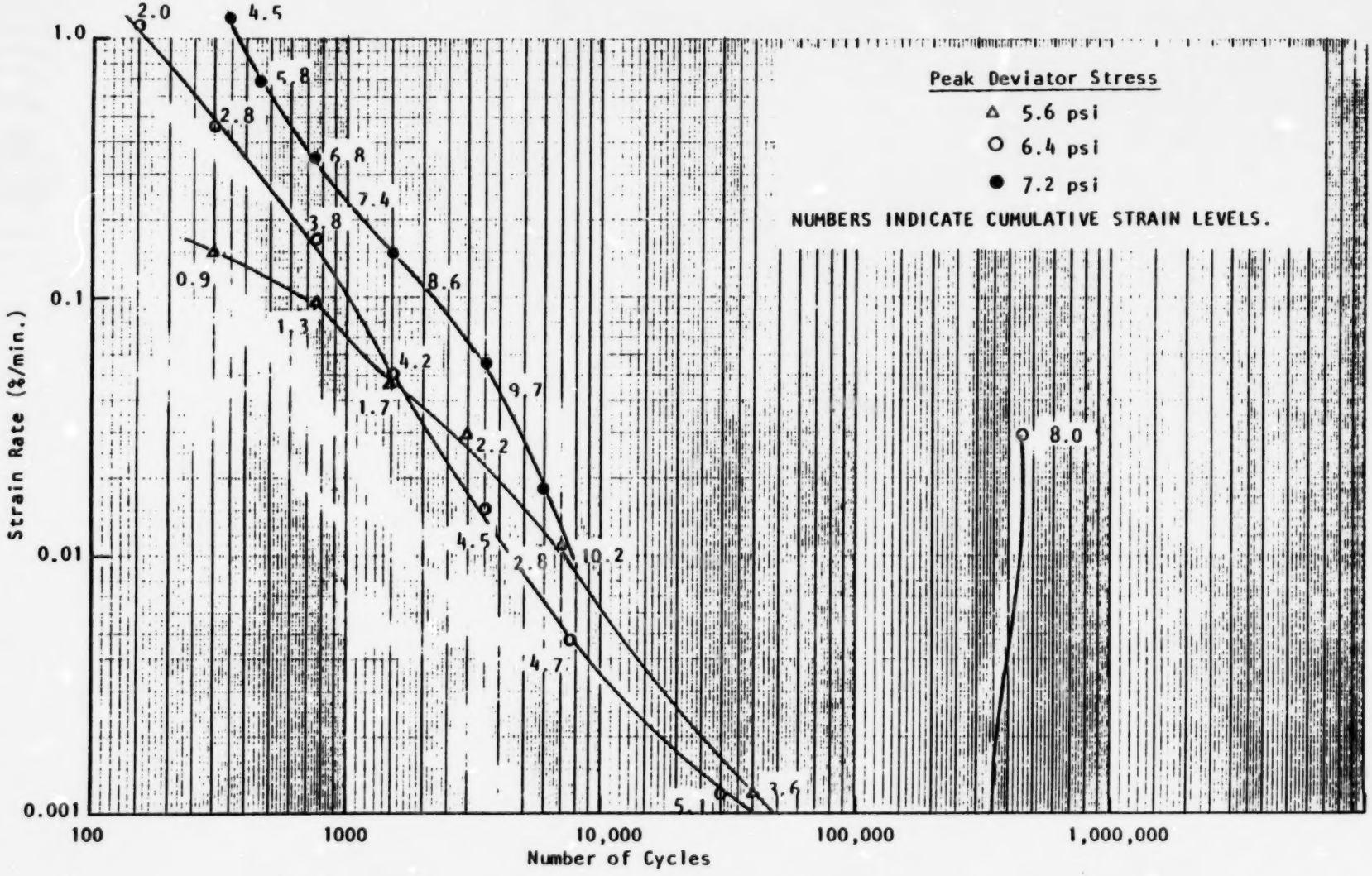


FIGURE NO. 15: RELATIONSHIP BETWEEN AXIAL STRAIN RATE AND NUMBER OF LOAD CYCLES FOR SPECIMEN FROM COMPOSITE SAMPLE "A".

DYNAMIC TRIAXIAL TEST RESULTS FOR SPECIMENS FROM COMPOSITE SOIL "B"

Table 3 shows the test parameters investigated to date for specimens prepared from composite soil "B". Figure No. 16 shows the development of cumulative strain with number of load cycles. Insufficient data is currently available to define a threshold stress, but it would appear that it will be less than the 6 psi identified for soil "A". Figure No. 17 suggests that the threshold strain levels are lower by at least 0.1% as compared to soil "A". Pore pressure trends established for soil "A" seem to be reflected in soil "B" as shown in Figure No. 18. Figure No. 19 shows that the dynamic modulus of these specimens which are showing rapid accumulation of strain are higher than the modulii for comparable strain performance in soil "A". This is a function of the reduced elastic strain levels shown in Figure No. 17.

TABLE 3: DYNAMIC TRIAXIAL TEST PARAMETERS FOR SPECIMENS PREPARED FROM COMPOSITE SAMPLE "B".

TEST NUMBER	SOIL TYPE	TEST CONDITION	CONSOLIDATION PRESSURE (psi)	BACK PRESSURE (psi)	PEAK CYCLIC DEVIATOR STRESS (psi)	NO. OF CYCLES	CUMULATIVE STRAIN (%)	RANGE OF ELASTIC STRAIN (%)	RANGE OF DYNAMIC MODULUS (psi)	RANGE OF PORE PRESSURE (psi)	INITIAL WATER CONTENT (%)	FINAL WATER CONTENT (%)	DRY DENSITY (lbs./cu.ft.)	REMARKS
12	B	Undrained	2.0	35	6.6	1500	13.8	0.45→ 0.39	1450→ 1700	+.48→ -.10	28.4	37.5	90.19	Sample Failed
14	B	Undrained	2.0	40	6.6	6200	1.6	0.22→ 0.27	3000→ 2450	+.2→ +.72	28.5	36.3	91.43	Sample Failed
					7.2	1300	13.2	0.45→ 0.38	1550→ 1900	+.4→ -.56				

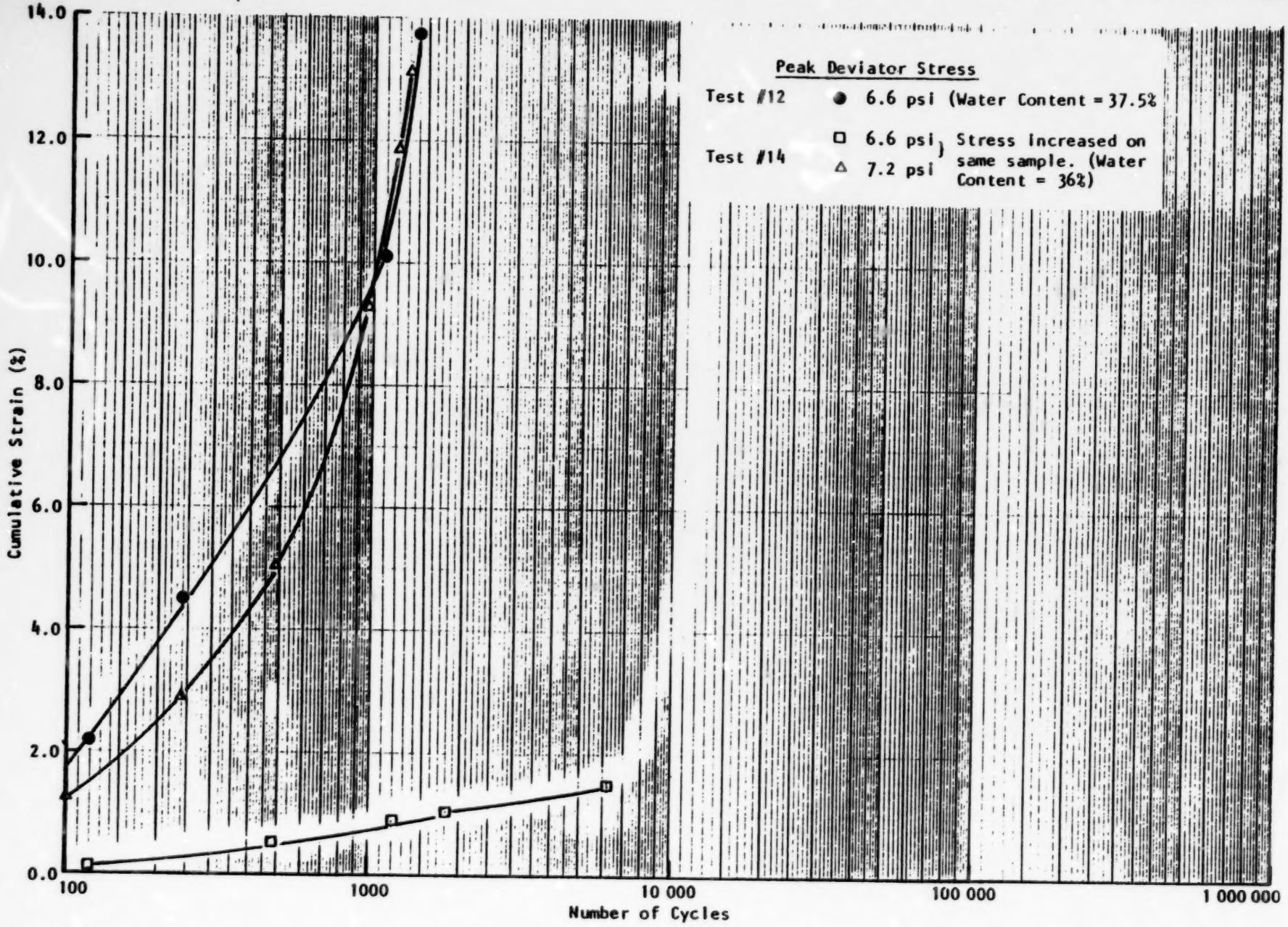


FIGURE NO. 16: RELATIONSHIPS BETWEEN CUMULATIVE AXIAL STRAIN AND NUMBER OF LOAD CYCLES FOR SPECIMENS FROM COMPOSITE SAMPLE "B".

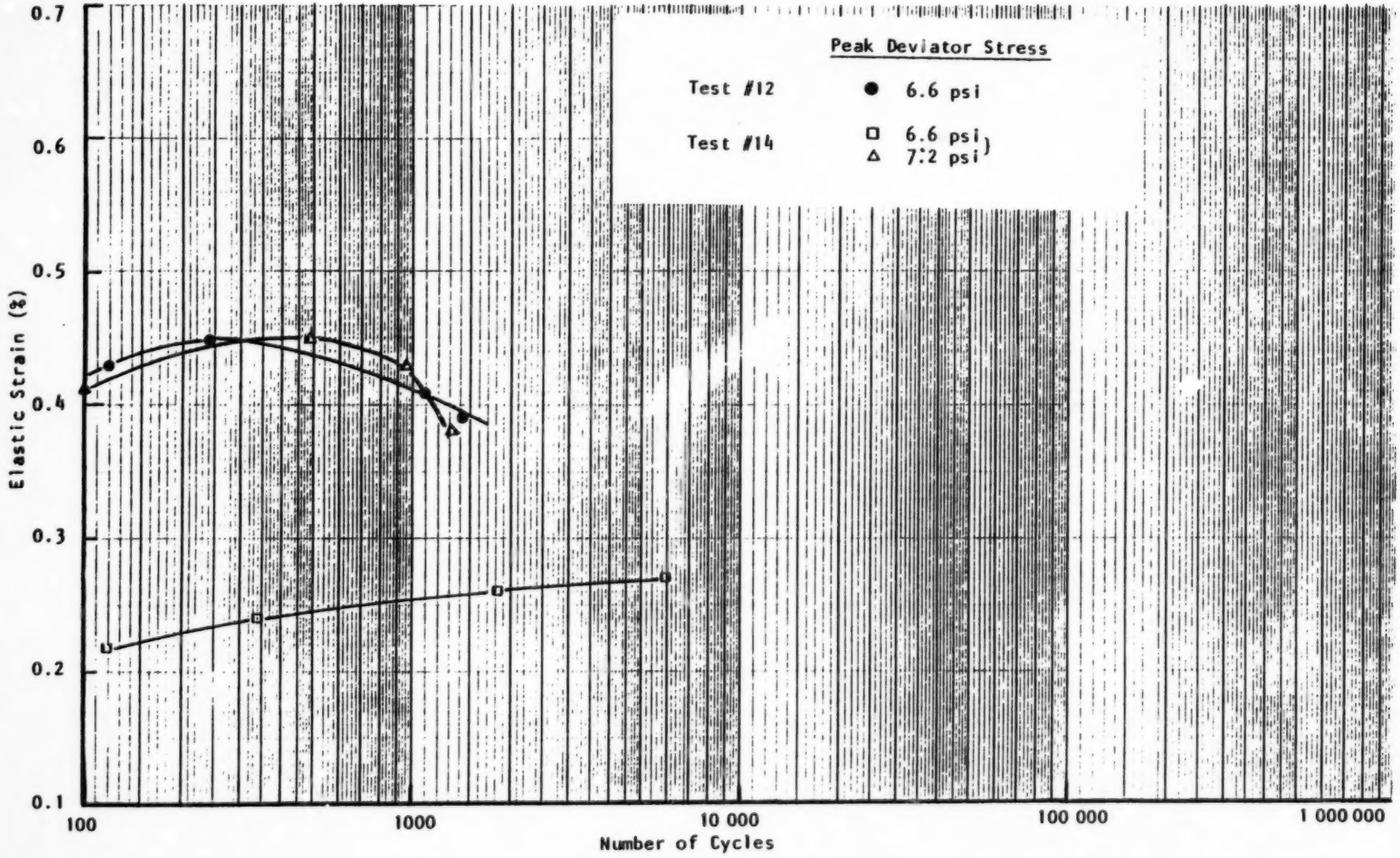


FIGURE NO. 17: RELATIONSHIPS BETWEEN AXIAL ELASTIC STRAIN AND NUMBER OF LOAD CYCLES FOR SPECIMENS FROM COMPOSITE SAMPLE "B".

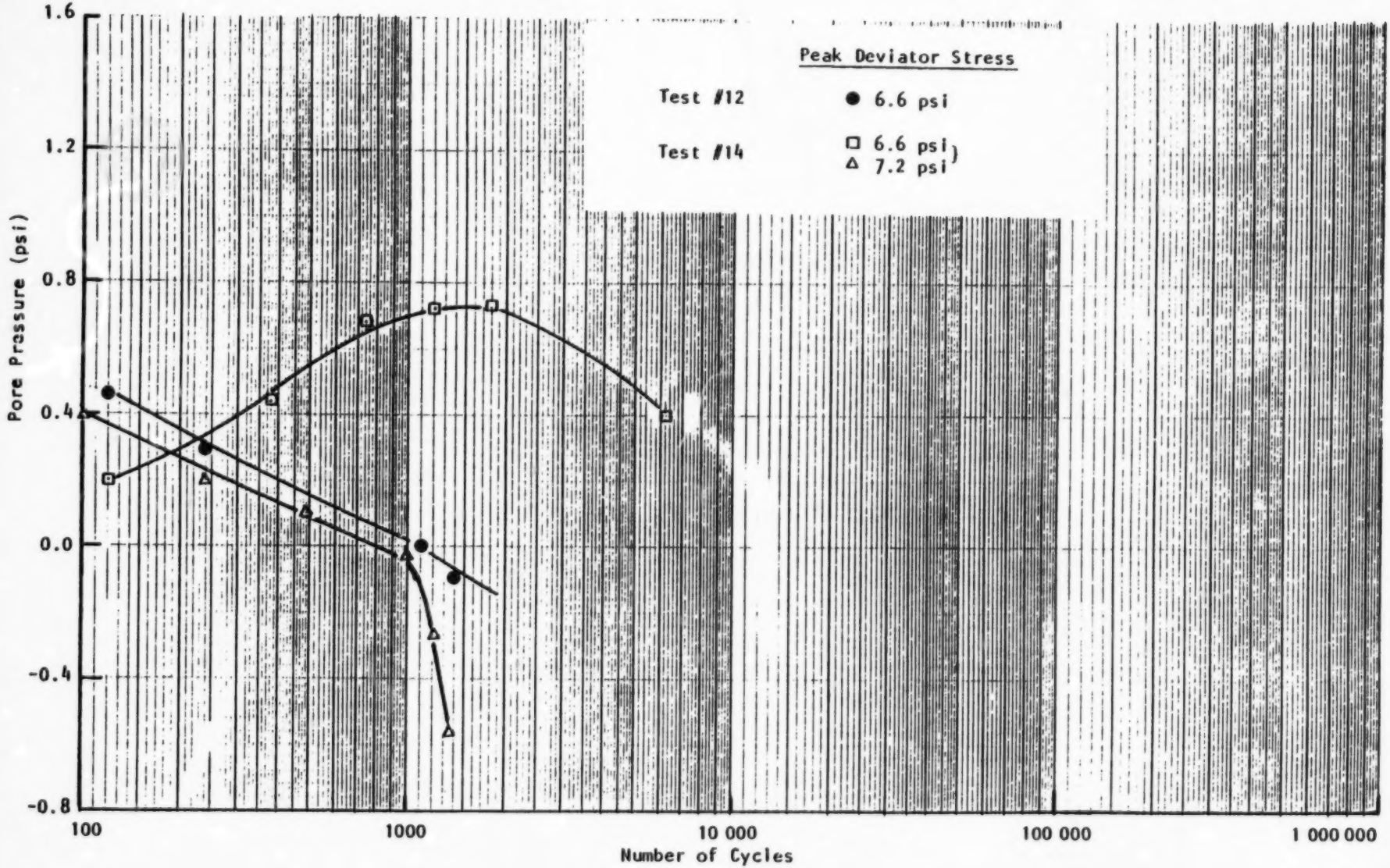


FIGURE NO. 18: VARIATION IN PORE PRESSURE WITH NUMBER OF LOAD CYCLES FOR SPECIMENS FROM COMPOSITE SAMPLE "B".

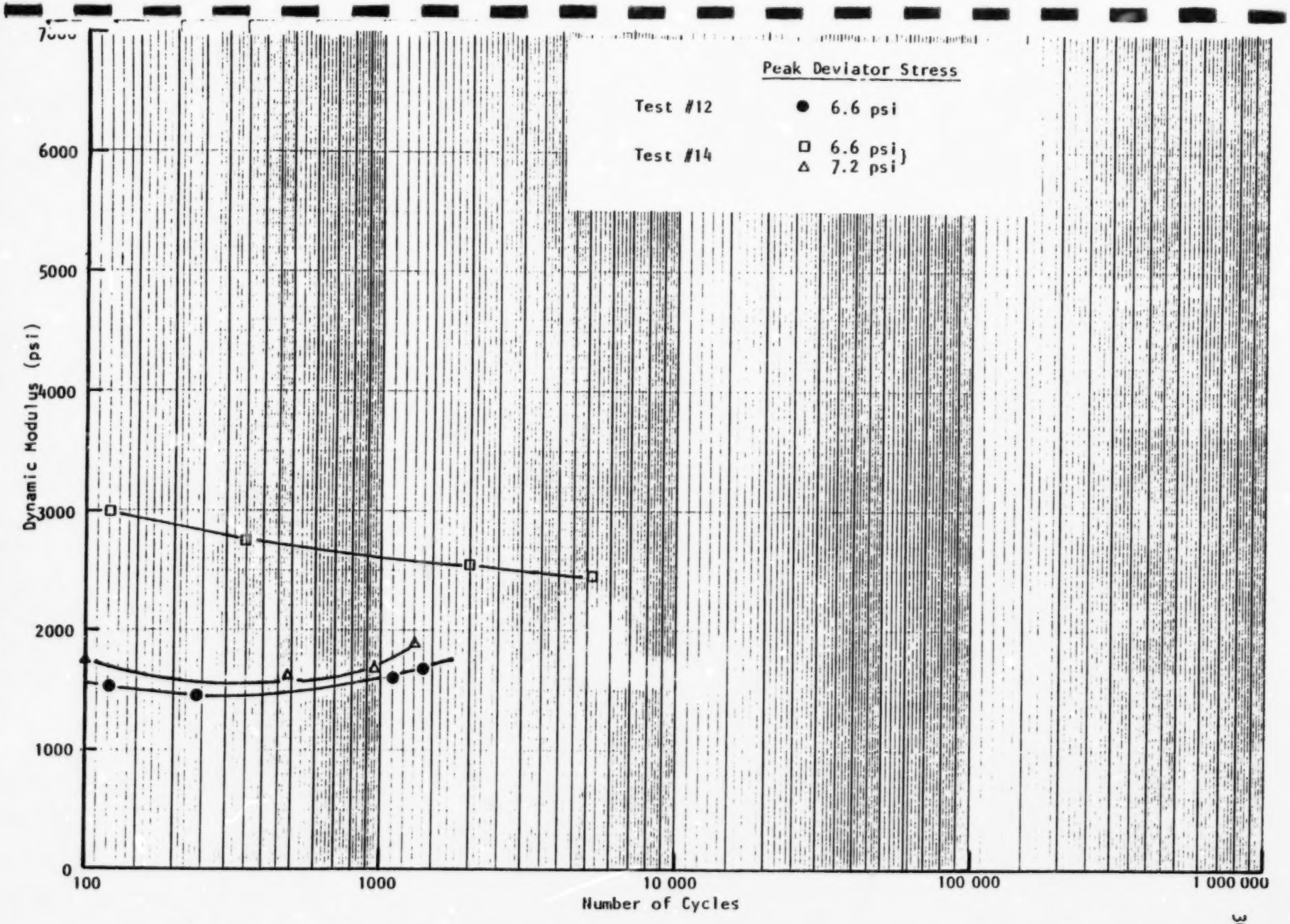


FIGURE NO. 19: VARIATION IN DYNAMIC MODULUS WITH NUMBER OF LOAD CYCLES FOR SPECIMENS FROM COMPOSITE SAMPLE "B".

CONCLUSIONS FROM LABORATORY TEST PROGRAM

1. The basic behavior of the composite soils appears to follow the trends assumed in the British Rail* method. Composite soil "A" (Test Pits 2A, 14 and 16) experiences rapid cumulative strains above dynamic stress levels ($\sigma_1 - \sigma_3$) of about 6 psi and elastic strain levels above 0.5%. Soil "B" (Test Pits 9 and 25) behavior suggests that lower dynamic stress levels are required to cause the same development in cumulative strain, based on limited test data. However, the threshold elastic strain levels are smaller than for soil "A" even though the clay contents and water contents are higher in soil "B".

2. The cumulative strain rates at approximately 10,000 load cycles are very similar, in contrast to the British Rail results, and the strain rate may "suddenly" increase resulting in unexpected large strains at the 1,000,000 cycle level.

RECOMMENDATIONS REGARDING FURTHER TESTING

1. Two dynamic tests are required on composite sample "B" to clearly define threshold stress and elastic strain levels. This would allow an assessment of the influence of clay content on threshold levels by comparison of composite soil "A" and "B".

2. At least two tests should be conducted on soil "A" at a confining pressure of 4 psi to simulate greater depth granular layer.

3. Three tests should be conducted on soil "A" at stress levels below the threhold level with full drainage provided. The real field behavior would lie between the fully drained and undrained cases.

*See Bibliography,

RECOMMENDATIONS REGARDING BRITISH RAIL DESIGN METHOD

The fundamental assumptions of the British Rail method that there exists a threshold stress level below which cumulative strain rates are very low after an initial "seating" period, and that this threshold stress should not be exceeded at any point in the subgrade in the field by the heaviest wheel load appears to be conservative. Significant reductions in predicted ballast thicknesses may be realized by designing to allow plastic strains to accumulate at a rate which is consistant with maintainence schedules dictated by ballast or rail parameters.

An attractive approach to this problem is to develop a "cumulative damage" relationship which predicts the strain rate by taking into account the previous load distribution history. Laboratory tests which subject specimens to realistic stress pulse shapes (as opposed to the currently used sine function) and pulse amplitude distribution (to simulate actual wheel load distributions) can easily be performed and would provide the material behavior parameters. Design charts similar to those proposed by the British Rail could then be constructed. These would necessarily incorporate assumptions regarding wheel load distribution and tolerable rates of cumulative strains in the subgrade.

RECOMMENDATIONS REGARDING FIELD TESTING

Field monitoring is an essential part of any effort to improve the conservative assumptions used in the British Rail method. Measurement of elastic strains and cumulative plastic strains as functions of the wheel load histories are required. These measurements are particularly important during even the passage of the first train since it is during this period when there is a great spread in cumulative strain rate as a function of wheel load. Direct measurement of elastic strain by permanently installed capacitance type strain

gauges (Bison) is recommended. Settlement plates, which can be monitored periodically by optical leveling techniques, would be adequate for plastic strain measurements. Periodic records of water table elevation, frost penetration, rainfall, etc. would be required for a proper interpretation of field data.

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